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PERFORMANCE-BASED STRUCTURAL FIRE DESIGN AND TESTING OF STRUCTURES

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ABSTRACT

Fire incident in buildings is common in Hong Kong and this could lead to heavy casualties due to its high population density, so the fire safety design of the framed structure is an important research topic. This paper describes a computer tool for determination of capacity of structural safety against various fire scenarios and the well-accepted second-order direct plastic analysis is adopted for simulation of material yielding and buckling. A computer method is developed to predict structural behaviour of bare steel framed structures at elevated temperatures but the work can be applied to structures made of other materials. These effects of thermal expansion and material degradation due to heating are required to be considered in order to capture the actual behavior of the structure under fire. Degradation of material strength with increasing temperature is included by a set of temperature-stress-strain curves according to BS5950 Part 8 mainly, which implicitly allows for creep deformation. Several numerical and experimental verifications of framed structures are presented and compared against solutions by other researchers. The proposed method allows us to adopt the truly performance-based structural fire analysis and design with significant saving in cost and time.

KEYWORDS: *Fire analysis; Finite element method; Steel structures; Nonlinear analysis*

1. INTRODUCTION

In past decades or so, the fire safety of the buildings or an individual component was comprehensively evaluated by the fire test, i.e. as shown in the Fig. 1, which is a fire laboratory in Hong Kong. Especially, fire test is essential to rationalize the intricate behaviour of the structure due to various fire effects despite its higher cost. Subsequently, the fire design specifications, such as BS5950 Pt. 8, (2000) and Eurocode 3 Pt. 1.2, (1993), was established, by which can assess the fire resistance of structural components under the international standardization for organization (ISO 834, (1985)). Approximate and linearised structural behaviour used the component check when using the traditional fire safety design codes. In reality, the individual components are only parts of a structure and the interaction of members needs to be considered for an accurate solution. The cooler parts of a structure under fire may support and restraint the components weakened by elevated temperature. To produce a safe design economically, the complete structure should be therefore considered holistically. Nonlinear computer software for fire analysis can be used for the fire safety design of a complete structure allowing for different thermal effects and associated nonlinear structural effects. In addition, the fire safety design of the large-scale building structures is needed to be used with the simulation-based approach. The performance-based fire safety design approach is worthy of further research and development, which attributes to cost saving, enhanced safety level and efficient design. In most cases, the interaction among the heated and cool members on the complete structural system under fire scenarios is complicated and cannot be considered in the prescriptive fire safety design. Thus, the advanced numerical fire analysis for the future fire safety design is imperative and proposed in this paper using proper second-order analysis software



Figure 1: Fire chamber of a fire laboratory in Hong Kong

Jeanes (1982,1985) presented the fire analysis including two-dimensional element for simulating the floor system under fire. In this study, members in the frame are modeled by the beam-column elements and the non-conforming triangular plate bending element is used to model the floor slab. However, this analysis requires great computational effort and thus it is unsuitable for the fire safety design of practical large-scale building structures.

In the continental Europe, researchers such as Dotreppe et al. (1982) and the ARBED Research Center (1982,1992) studied the structural behaviour of beams, columns and frames under fire by the finite element method. Schleich et al. (1986) developed the nonlinear fire analysis using the incremental-iterative Newton-Raphson solution procedure to model nonlinearities. Franssen (1987) presented a fire analysis which considers non-uniform temperature distribution, material yielding and geometric nonlinearities. Franssen (1990) modified the constitutive law including loading-unloading response and developed the computer program SAFIR (Franssen et al., (2000)) based on a fine grid of elements over each cross-section. Using this computer program SAFIR, Lim et al. (2004a, 2004b) investigated the fire behaviour of two-dimensional reinforced concrete slab under fire. Vila Real et al. 2004 presented a comparative study between numerical results by SAFIR 2000 and fire design code - Eurocode 3 (1993).

In the U.K., Saab and Nethercot (1991) presented the nonlinear structural analysis of plane frames subject to fire. Their basic numerical formulation of the nonlinear solution at ambient condition was based on the research by El-Zanaty and Murray (1983) based on which Najjar and Burgess (1996) further utilized their previous works to develop the fire analysis software VULCAN. Similarly, Bailey (1998) made use of this work to develop the computer program 3DFIRE. This basic nonlinear formulation allows the modeling of semi-rigid connections, lateral-torsional buckling, continuous floor slabs and strain reversal. Huang et al. (1999) extended their previous work further to incorporate the two-dimensional finite element in their fire analysis in order to model the behaviour of reinforced concrete slabs under fire. Later Huang et al. (2000) carried out a nonlinear three-dimensional fire analysis using VULCAN for composite framed structure. This shows computer analysis is a well-accepted tool for checking of structural safety under fire.

Liew et al. (1998) presented the performance-based and nonlinear fire analysis of the steel framed structures. Material yielding behaviour is considered by plastic hinges formed at the element ends and the mid-span. The member is then divided into two elements when a plastic hinge at the mid-span. Recently, Ma and Liew (2004) further developed their plastic hinge approach on their fire analysis to study three-dimensional

bare steel frames under fire. In the paper, Liew (2004) provided an overall view on performance-based code and the approaches for designing steel structures in fire considering a multi-dimensional integration of fire simulation, heat transfer analysis, emergency evacuation and structural resistance. Various fire modes and heat transfer analysis methods are presented and used to model the large deflection and plasticity using nonlinear analysis.

Iu and Chan (2004) presented a nonlinear fire analysis by the plastic hinge approach where all thermal effects are incorporated into the stiffness formulation of the cubic finite element. Later, Iu et al. (2005) extended the fire analysis to consider the structural effects in the cooling phase in which the plastic strains are formulated in the stiffness formulation to account for permanent deformations during cooling. The plastic hinge method on the fire analysis was refined by Iu et al. (2007) to allow for the axial residual strength of the member after yielding, which is important to model the catenary action when a beam experiences significant large deflection.

Simulation-based nonlinear fire analysis is an important development of the fire safety design for the framed structures, especially for the large-scale steel structures because of its complexity under fire. To this end, this paper presents a fire study of the bare steel frames using structural analysis software NIDA (2007) to tackle the nonlinear behaviour of the framed structures exposed to fire. Its numerical stiffness formulation is derived from the fifth-order displacement function, namely the PEP element (Chan and Zhou (1995)), which can simultaneously satisfy both equilibrium and compatibility conditions at midspan of an element. The solution of nonlinearities is resorted to the incremental-iterative solution procedure, such as the Newton-Raphson method. Based on a set of temperature-stress-strain curves to BS5950 Part 8 (2000), material degradation due to rising temperature is taken into consideration by using the equivalent section properties. The thermal expansion is also incorporated in the thermal analysis of NIDA (2007). In dealing with the effect of material nonlinearity, the secant elastic modulus at the various temperatures is directly used in the analysis software NIDA (2007). This complete analysis procedure can be carried out in any nonlinear structural analysis software without modifying the source code, which implies that engineers can do the same for his structures using his familiar non-linear structural analysis software.

2. BASIC NUMERICAL FORMULATION OF PEP ELEMENT

According to the finite element approach, the basic stiffness formulation in the present analysis is based on the fifth-order displacement function to form the self-equilibrium element namely as the PEP element (Chan and Zhou (1995)) (Point-wise equilibrium polynomial). The main feature of this PEP element is to consider the member bowing

effect implicitly in the element stiffness formulation, so a single element can be used to predict the flexural buckling behaviour of a member.

The second-order effects due to member bowing and flexural bending are introduced in the internal strain energy expression in Eq. (1) by inclusion of second-order terms. The total potential energy for nonlinear analysis of beam-column element under normal temperature can therefore be written as:

$$U = \frac{EA}{2} \int_{-L/2}^{L/2} \left(\frac{du}{dx} \right)^2 dx + \frac{EI_z}{2} \int_{-L/2}^{L/2} \left(\frac{d^2v}{dx^2} \right)^2 dx + \frac{P}{2} \int_{-L/2}^{L/2} \left(\frac{dv}{dx} \right)^2 dx + \frac{EI_y}{2} \int_{-L/2}^{L/2} \left(\frac{d^2w}{dx^2} \right)^2 dx + \frac{P}{2} \int_{-L/2}^{L/2} \left(\frac{dw}{dx} \right)^2 dx \quad (1)$$

The external work done, V , is equal to the applied force multiplied by the corresponding displacement as follows:

$$V = \{d\}^T \{f\} \quad (2)$$

in which u , v and w are the axial deformation and lateral displacements in the direction in y -axis and z -axis, respectively. EA and EI are the axial rigidity and flexural rigidity about corresponding axes, respectively. P is the axial member load. And $\{d\}$ and $\{f\}$ are the column vectors of the displacement and external applied force with respect to the corresponding degrees of freedom. $\{d\}$ is a displacement column vector given by $\langle u_1, u_2, v_1, v_2, w_1, w_2, \theta_{1x}, \theta_{2x}, \theta_{1z}, \theta_{2z}, \theta_{1y}, \theta_{2y} \rangle^T$.

The total potential energy functional can be expressed as the sum of the strain energy and the external work done as,

$$\Pi = U - V$$

$$\Pi = \frac{EA}{2} \int_{-L/2}^{L/2} \left(\frac{du}{dx} \right)^2 dx + \frac{EI_z}{2} \int_{-L/2}^{L/2} \left(\frac{d^2v}{dx^2} \right)^2 dx + \frac{P}{2} \int_{-L/2}^{L/2} \left(\frac{dv}{dx} \right)^2 dx + \frac{EI_y}{2} \int_{-L/2}^{L/2} \left(\frac{d^2w}{dx^2} \right)^2 dx + \frac{P}{2} \int_{-L/2}^{L/2} \left(\frac{dw}{dx} \right)^2 dx - \{d\}^T \{f\} \quad (3)$$

By the principle of stationary total potential energy, the equilibrium condition and thus the stiffness coefficients can be obtained by taking the first variation of the functional as,

$$\delta \Pi = \frac{\partial \Pi}{\partial u_i} + \frac{\partial \Pi}{\partial q} \frac{\partial q}{\partial u_i} = 0 \quad \text{For } i = 1, 2, 3 \quad (4)$$

The element matrix in three-dimensional space can be extended directly from the foregoing equations by considering the stiffness in the y - and z -axes as,

$$M_{1n} = \left[\frac{EI}{L} \right]_n [c_{1n}(\theta_{1n} + \theta_{2n}) + c_{2n}(\theta_{1n} - \theta_{2n})] \quad (5)$$

$$M_{2n} = \left[\frac{EI}{L} \right]_n [c_{1n}(\theta_{1n} + \theta_{2n}) - c_{2n}(\theta_{1n} - \theta_{2n})] \quad (6)$$

$$P = EA \left[\frac{e}{L} + \sum_{n=y,z} (b_{1n}(\theta_{1n} + \theta_{2n})^2 + b_{2n}(\theta_{1n} - \theta_{2n})^2) \right] \quad (7)$$

$$M_t = \frac{GJ + Pr^2}{L} \theta_t \quad (8)$$

in which GJ is torsional rigidity. M_t is torsional moment. θ_t is twist angle. r is polar radius of gyration. And $n = y, z$ - principal axes. The numeric subscript stands for node number. c_{1n} , c_{2n} , b_{1n} and b_{2n} are the incremental secant stiffness coefficient, which are given by Chan and Zhou (1995). e is axial deformation. θ is the element nodal rotations.

The tangent stiffness matrix relates the incremental change in forces to a corresponding change in displacements. This can be obtained by taking the first variation of the secant stiffness matrix with respect to the displacements at the degrees of freedom as,

$$\delta^2 \Pi = \frac{\partial^2 \Pi}{\partial u_i \partial u_j} \delta u_i \delta u_j = \left[\frac{\partial(\delta \Pi)}{\partial u_j} + \frac{\partial(\delta \Pi)}{\partial q} \frac{\partial q}{\partial u_j} \right] \delta u_i \delta u_j \quad (9)$$

The tangent stiffness matrix can finally be determined as,

$$[K_T] = \sum_{Nele} [L][K_T][L]^T = \sum_{Nele} [L]([T]^T [K_e][T] + [N])[L]^T \quad (10)$$

in which $[T]$ is transformation matrix relating the member basic forces to the element force in the local coordinate. $[L]$ is local to global transformation matrix. And $[N]$ is a matrix to account for the stiffness change due to the initial force and the translational displacements.

3. NONLINEAR SOLUTION PROCEDURES

A nonlinear incremental-iterative procedure is used in this study to trace the nonlinear equilibrium path due to the geometric nonlinearities i.e. the P- Δ and P- δ effects. An incremental-iterative solution procedure is developed to cope with either the constant load increment of Newton-Raphson method or varied load increment, such as the arc-length method by Crisfield (1981) or the minimum residual displacement method by Chan (1988). The solution procedure of constant load Newton-Raphson method is schematically shown in Fig. 2 at temperature $T^\circ C$, which is basically used in the present approach.

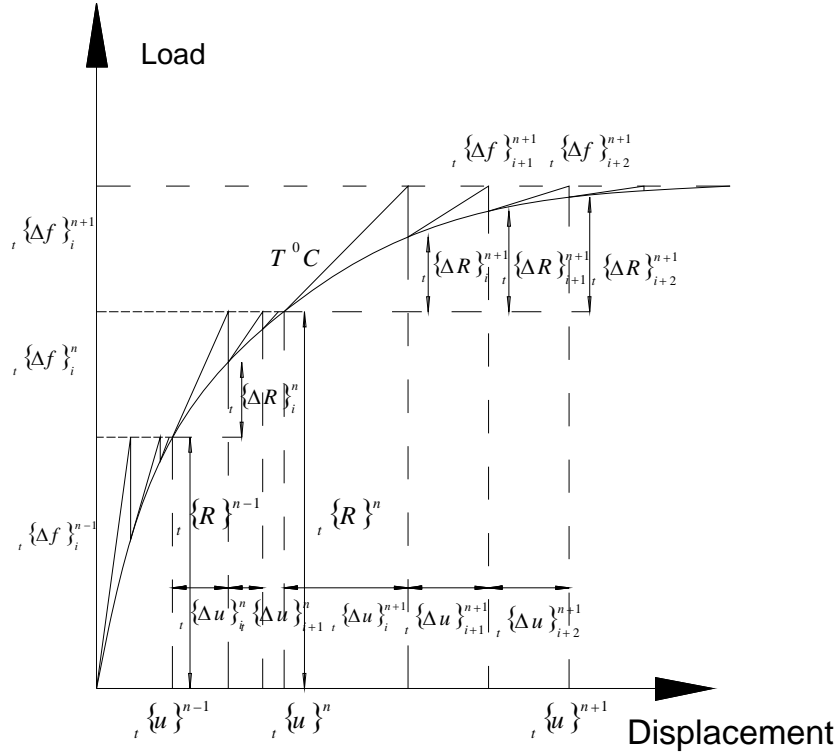


Figure 2: Incremental-iterative scheme of Newton-Raphson method at t -th thermal cycle

Based on Newton-Raphson method to formulate the nonlinear solution procedures, the simple compact matrix form of equilibrium equation at a particular t -th temperature level ($T^0 C$) can be expressed as,

$$\{f\} - [K_s]\{u\} = [K_T]\{\Delta u\} \quad (11)$$

in which $[K_s]$ is secant stiffness which represents the internal member resistances. And $\{f\}$ is applied loading.

In the constant load Newton Raphson scheme, the incremental displacement can be determined by the tangent stiffness in Eq. (12), and the total displacement is obtained by accumulation of incremental displacements in Eq. (13). In the meantime, the structural geometry is revised according to the updated Lagrangian formulation.

$${}_t\{\Delta u\}_{i+1}^n = {}_t[K_T(\{R\}_i^n)]^{-1} {}_t\{\Delta f\}_i^n \quad (12)$$

$${}_t\{u\}_{i+1}^n = {}_t\{u\}_i^n + {}_t\{\Delta u\}_{i+1}^n \quad (13)$$

in which ${}_t[K_T(\{R\}_i^n)]$ is the incremental tangent stiffness at $\{R\}_i^n$. ${}_t\{\Delta f\}_i^n$ is the unbalanced force between external applied load and internal member resistance. The incremental displacements in global coordinates can be transformed to the member deformations ${}_t\{\Delta u_e\}_{i+1}^n$ by multiplying the transformation matrix $[L]^T$. ${}_t\{\Delta R_e\}_{i+1}^{n+1}$ can then be evaluated in local coordinates as,

$${}_t\{\Delta u_e\}_{i+1}^n = [L]^T {}_t\{\Delta u\}_{i+1}^n \quad (14)$$

$${}_t\{\Delta R_e\}_{i+1}^n = [K_s({}_t\{R\}_i^n)] {}_t\{\Delta u_e\}_{i+1}^n \quad (15)$$

in which $[K_s({}_t\{R\}_i^n)]$ is the incremental secant stiffness at ${}_t\{R\}_i^n$. After determination of the member resistant forces, the incremental member resistance is accumulated to the total member resistance in global coordinates as,

$${}_t\{R\}_{i+1}^n = {}_t\{R\}_i^n + [T][L] {}_t\{\Delta R_e\}_{i+1}^n \quad (16)$$

in which $[T]$ is the transformation matrix from member coordinate to global coordinate. The unbalanced force ${}_t\{\Delta f\}_{i+1}^n$ is therefore obtained as,

$${}_t\{\Delta f\}_{i+1}^n = {}_t\{f\}^n - {}_t\{R\}_{i+1}^n \quad (17)$$

The above incremental-iterative procedure is repeated for a specified temperature level ($T^\circ C$) until equilibrium point is achieved at specified applied load level. The applied load is kept constant throughout the heating sequence. The next thermal cycle ($(T+1)^\circ C$) starts with the same incremental-iterative procedure, when the constant load Newton-Raphson method is utilized as Eqs. (11)-(17), which is graphically shown in Fig. 3. The process is repeated throughout the thermal history until a numerical divergence is detected.

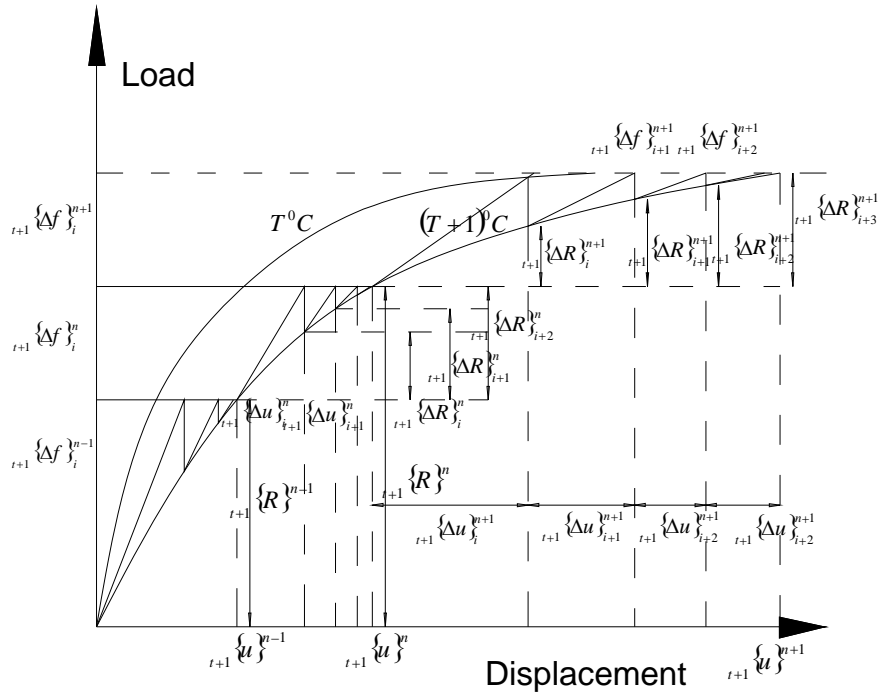


Figure 2: Incremental-iterative scheme of Newton-Raphson method at $(t+1)$ -th thermal cycle

4. THERMAL ANALYSIS OF PEP ELEMENT

The thermal effects considered in the present fire analysis, mainly consist of thermal expansion and material degradation due to heating, where the thermal creep effect is only implicitly considered by a set of temperature-stress-strain curves. With respect to thermal expansion, once the temperature distribution across the section has been determined, the change of the member length ΔL resulting from rising temperature ΔT can be evaluated as Eq. (18) based on the coefficient of thermal expansion coefficient α , taken as 12×10^{-6} . The change of the member length can be converted to the nodal deformations $\{\Delta u\}$ of a member in its axial directions. This deformed geometry of a structure due to rising temperature is treated as an initial geometry of the structure for subsequent second-order analysis. The change of geometry of the whole structural system can then be updated in accordance with the updated Lagrangian formulation in Eq. (19) as,

$$\Delta L_i = \alpha \Delta T \times L \quad (18)$$

$${}_t\{u\}_0 = \{u\}_0 + {}_t\{\Delta u\} \quad (19)$$

The thermal effect on the physical properties of steel due to material degradation is considered in the reduction of the elastic modulus E_T and effective yield strength σ_{yT} in accordance with BS5950 Pt. 8 (2000) and tabulated in Table 1. According to the present fire approach, material degradation is taken into account through the equivalent deteriorated section properties, including area A_T , moment of inertia I_T and plastic section modulus Z_{pT} , to model the temperature variation effect in the physical properties of an element as (Landesmann (2005)).

Table 1: Expressions of effective yield stress and elastic modulus for stress-strain-temperature curves

Temperature ($^{\circ}C$)	Yield stress, $\sigma_{yT} = \psi_T \sigma_{y20}$	Elastic modulus, $E_T = \psi_T E_{20}$
$80^{\circ}C < T \leq 400^{\circ}C$	$\sigma_{y20} \left(0.978 - 0.034 \frac{T}{350} \right)$	$E_{20} \left[1 - 2.8 \left(\frac{T - 20}{1485} \right)^2 \right]$
$400^{\circ}C < T \leq 550^{\circ}C$	$\sigma_{y20} \left(1.553 - 0.155 \frac{T}{100} \right)$	$E_{20} \left[1 - 2.8 \left(\frac{T - 20}{1485} \right)^2 \right]$
$550^{\circ}C < T \leq 600^{\circ}C$	$\sigma_{y20} \left(2.34 - 0.22 \frac{T}{70} \right)$	$E_{20} \left[1 - 3 \left(\frac{T - 20}{1463} \right)^2 \right]$

$$\begin{array}{l}
600^{\circ}C < T \leq 690^{\circ}C \\
690^{\circ}C < T \leq 800^{\circ}C
\end{array}
\left| \begin{array}{l}
\sigma_{y20} \left(1.374 - 0.078 \frac{T}{50} \right) \\
\sigma_{y20} \left(1.12 - 0.128 \frac{T}{100} \right)
\end{array} \right|
\left| \begin{array}{l}
E_{20} \left[1 - 3 \left(\frac{T-20}{1463} \right)^2 \right] \\
E_{20} \left[1 - 3 \left(\frac{T-20}{1463} \right)^2 \right]
\end{array} \right|$$

The equivalent section properties of area A_T and moment of inertia I_T is evaluated from the stiffness deterioration (i.e. E_T) due to heating, whereas the equivalent plastic section modulus Z_{pT} is based on the strength deterioration (i.e. σ_{yT}) with increasing temperature. The finite element mesh across the steel section displayed in Fig. 4 is used to determine the equivalent section properties of a whole section, which depends on the temperature distributions on each segment. Based on this segmentation process, an equivalent cross section area A_T can be obtained by summation of the reduced area of each segment as Eq. (21), which is based on effective axial stiffness $E_T A$ of the each segment shown in Fig. 4. To follow a similar methodology, an equivalent moment of inertia I_T can be obtained by Eq. (23) based on the effective bending stiffness $E_T I$ of each segment. In this calculation, the reduction factor ψ_T of elastic modulus E_{20} in Table 1 is used to convert the section properties (i.e. A and I) at ambient condition into equivalent section properties (i.e. A_T and I_T) at elevated temperatures. It is interesting to note that the centroid of the section is assumed to remain unchanged against the temperature variation across the section. Also this methodology is only applicable to doubly symmetric steel sections.

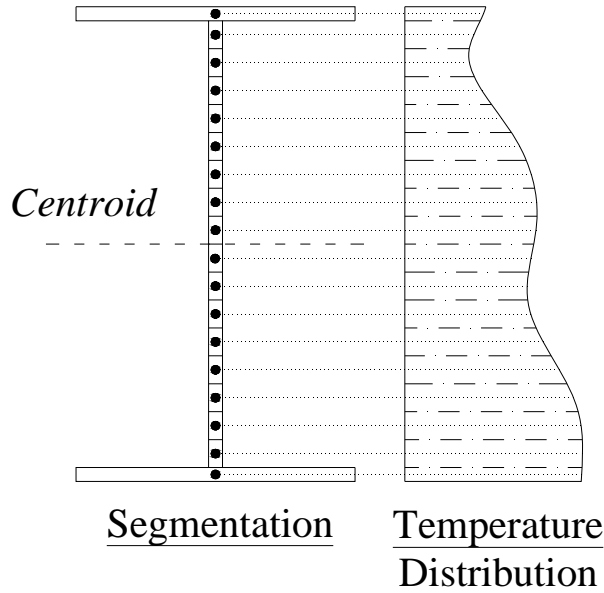


Figure 4: Segmentation of the section under arbitrary temperature distribution

$$E_{20} A_T = \int_A E_T dA = \int_A E_{20} \psi_T dA = E_{20} \sum_i^n \psi_{T,i} A_i \quad (20)$$

$$A_T = \sum_i^n \psi_{T,i} A_i \quad (21)$$

$$E_{20} I_T = \int_A E_T y^2 dA = \int_A E_{20} \psi_T y^2 dA = E_{20} \sum_i^n (\psi_{T,i} I_i + \psi_{T,i} A_i y_i^2) \quad (22)$$

$$I_T = \sum_i^n (\psi_{T,i} I_i + \psi_{T,i} A_i y_i^2) \quad (23)$$

in which T is temperature level. ψ_T is the reduction factor due to material degradation of both yield stress σ_{20} and elastic modulus E_{20} . σ_{y20} and E_{20} are respectively the yield stress and elastic modulus at ambient condition.

Similarly, an equivalent plastic section modulus Z_{pT} can be evaluated by using the reduction factor ψ_T for effective yield stress σ_{20} as,

$$\sigma_{y20} Z_{pT} = \int_A \sigma_{yT} y dA = \int_A \sigma_{y20} \psi_T y dA = \sigma_{y20} \sum_i^n (\psi_{T,i} A_i y_i) \quad (24)$$

$$Z_{pT} = \sum_i^n (\psi_{T,i} A_i y_i) \quad (25)$$

where i is the segment number. I_i is the moment of inertia of i -th segment about its centroid. A_i is the cross section area of i -th segment. y_i is the distance from the center of an i -th segment to the centroid of the whole member section.

At elevated temperature, the material behaviour of steel deteriorates with drastic change from elastic to plastic domains is unlikely. An elliptic transition between elastic and plastic domains is assumed in the steel material at high temperature. The elliptical transition in a more pronounced nonlinear response of the structure increases with rising temperature. The effective yield stress at elevated temperature is therefore used to replace the ambient yield stress. With the effective yield stress, the strength of material still increases gradually as the residual strength. As for such nonlinear material behaviour, a set of secant elastic modulus at the corresponding temperature levels according to the E_T in Table 1 is proposed to replace the E of flexural rigidity in the stiffness formulation from Eqs. (5) to (7), which are used in order to allow for gradual yielding process at different temperature levels. The same concept is applied to tangent stiffness formulation as Eq. (10). Note that the replaced E can be input in the input data file for structural analysis software without changing the software source code, implying that engineers can perform a structural fire design using a commercial package easily.

It is noteworthy that the aforementioned methodology aims at assessing the material behaviour of an element section under uniform and non-uniform heating in line with the plastic zone concept. Once the mean properties of the element section subjected to

rising temperatures are evaluated by virtue of this methodology, those properties can be incorporated into the plastic hinge formulation in order to study the nonlinear material behaviour of a structure, including gradual and full material yielding..

5. NUMERICAL VERIFICATIONS

The proposed nonlinear structural fire analysis is carried out using the structural analysis software NIDA (2007). To verify the present results of analysis, several steel framed structures under fire are selected for comparisons and verifications. Firstly, the experimental results of three small-scale steel frames under fire are studied. Secondly, the results of the compartment fire in three-storey steel frame are compared. Finally, a Cardington fire test of an eight-storey framed structure is used for verification.

5.1 Small-scale steel frame tests under uniform heating

Three small-scale steel frames with dimensions and loading conditions as depicted in Fig. 5 are heated uniformly in a series of tests performed by Rubert and Schaumann (1986). Also the numerical results of these small-scale frames from Iu and Chan (2004) are compared. There are a series of geometry for these steel frames, which are the inverted L-sharp frame (EHR), the single-bay portal frame (EGR) and the double-bay portal frame (ZSR). These frames are also composed of the same IPE80 I-section. The material yield stress at different temperature is shown in Table 1. The frames are also effectively restrained against out-of-plane deflection. The notations of measured deformations of the small-scale steel frames are also plotted in Fig. 5.

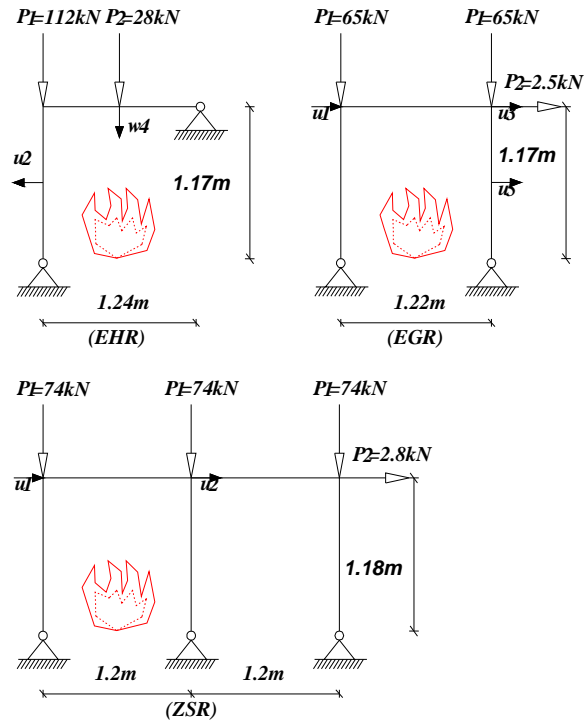


Figure 5: Geometries and loading applications of the small-scale steel frames

Figure 6 shows the deformations u_2 and w_4 of the EHR frame indicated in Fig. 5. The deformations from the present structural fire approach are consistent with the results from the fire tests Rubert and Schaumann (1986) and numerical results Iu and Chan (2004) in general. After 400°C , the significant nonlinear deformations occur, although the present fire approach can only predict the corresponding deformations with modest magnitude because, when the displacement is large, the curve in Fig. 6 involves much nonlinearity characterized by the steep load-displacement curve in the figure that small force difference results in a large discrepancy in displacement. In respect of w_4 in EHR, the mid-span deflection of beam decreases due to thermal expansion of column before 300°C . After 300°C , the deflection w_4 increases considerably, as the material yielding constitutes a larger deflection component than those by thermal expansion from heated column. The deformations u_2 and w_4 from the present fire approach are less than those from fire test Rubert and Schaumann (1986) and numerical results Iu and Chan (2004). It is principally due to the use of elastic modulus E_T as the secant elasticity at particular temperature may not rigorously predict the gradual plasticity.

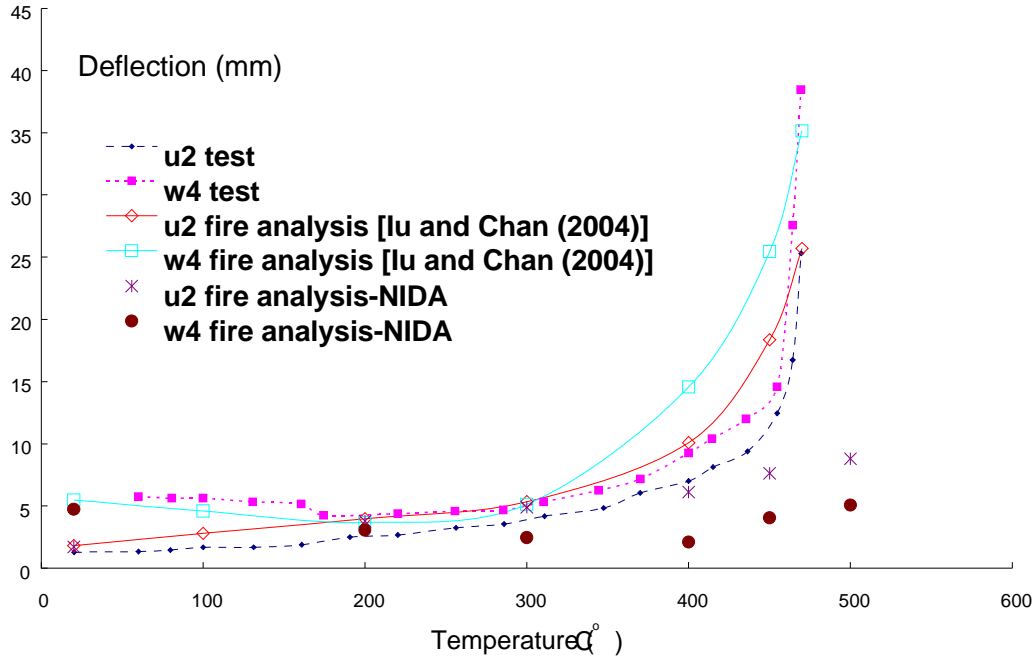


Figure 6: Temperature-deformation curves of the EHR frame

The corresponding displacements of the frames EGR and ZSR with rising temperature are also plotted in Figs. 7 and 8, respectively. These figures show that the displacements between the present analysis and both results Rubert and Schaumann (1986) and Iu et al. (2007) are in a good agreement until divergence. However, for ZSR frame, u_2 from present fire approach is much greater than those from experiment by Rubert and Schaumann (1986) and from numerical results by Iu and Chan (2004). It may be mainly due to the solution being too close to the divergent point at temperature level 530°C .

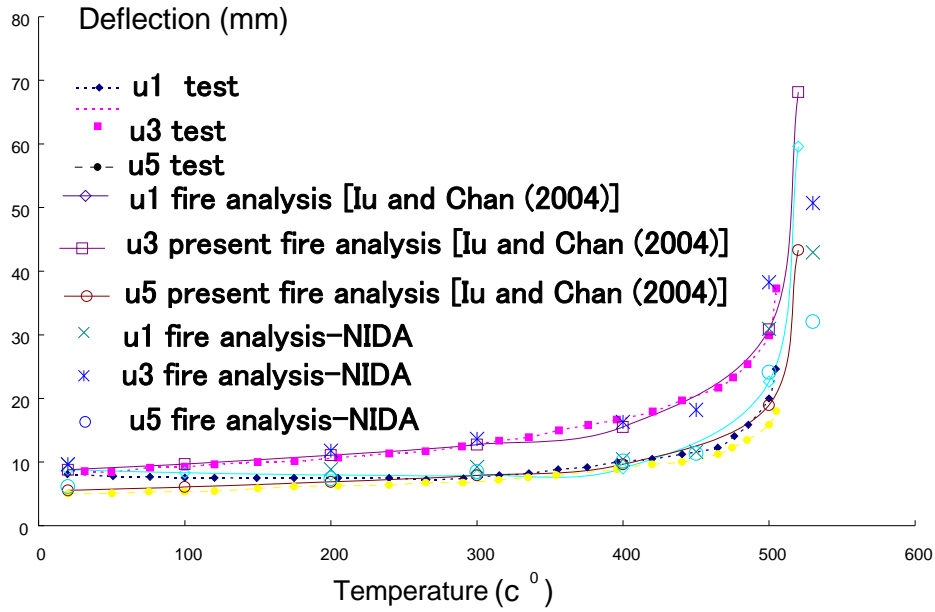


Figure 7: Temperature-deformation curves of the EGR frame

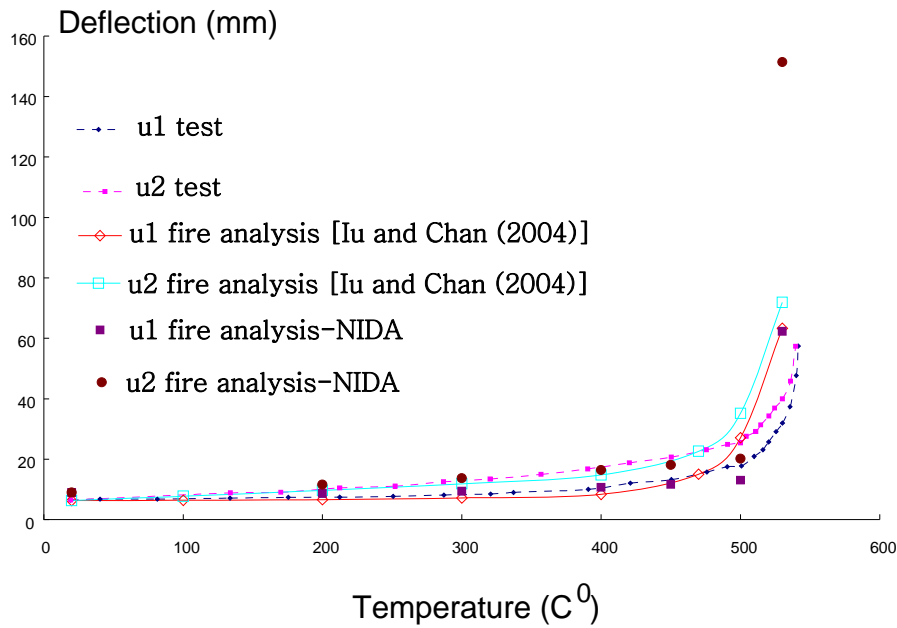


Figure 8: Temperature-deformation curves of the ZSR frame

5.2 Three-storey steel frame subject to compartment fire

Bailey (1998) numerically studied the behaviour of heated steel members in a three-storey steel frame shown in Fig. 9. A compartment fire is confined to bay 5 on the first floor in which the steel members are uniformly heated as indicated in Fig. 9. Properties and configuration of this frame and the applied loads of this numerical example are also shown in Fig. 9. All members are rigidly connected, and also the support conditions are fixed. In this verification, all members are modeled by the one element per member. The numerical results from Iu et al. (2007) are used for comparison.

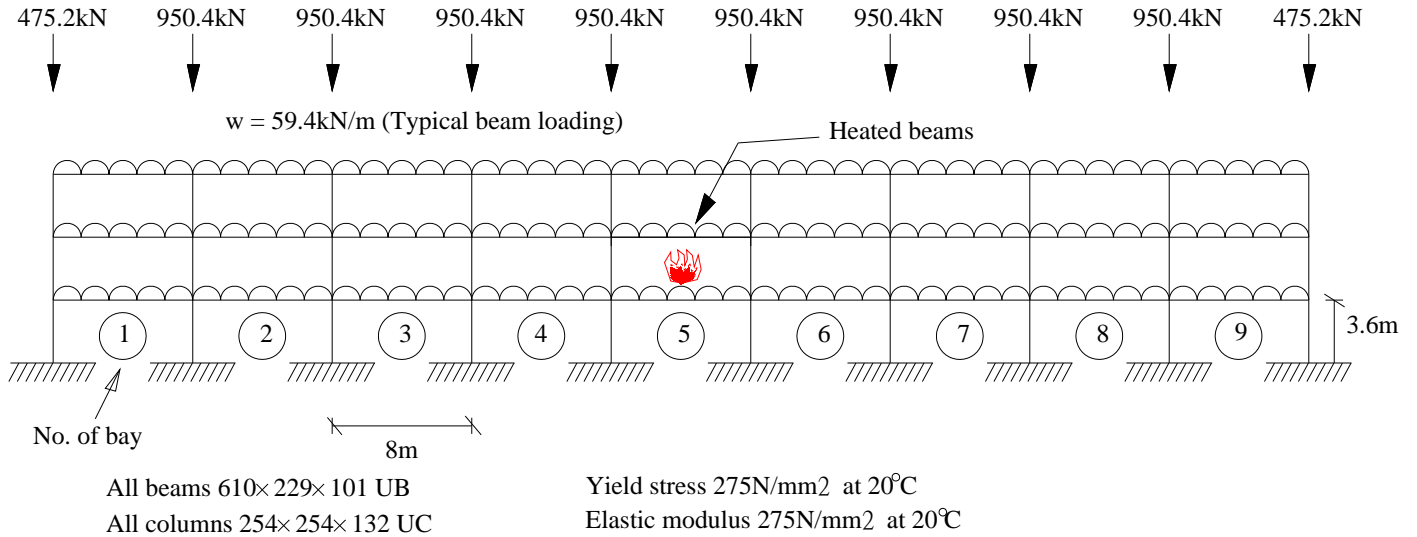


Figure 9: Geometric configuration and material properties of a three-storey steel frame

It should be noted that the structural response predicted from the present fire analysis is stiffer than the results by Bailey (1998), but in agreement with the results by Iu et al. (2005). According to the numerical results from present fire analysis as shown in Fig. 10, the material of the beam commences to yield gradually after 500°C. There is a considerable nonlinear deflection between 600°C and 606°C, as the elastic modulus E_T is reduced nearly by half at 600°C at which the steel material behaves more flexibly. The maximum mid-span deflection of heated steel beam is 85mm. As for the numerical results by Huang et al. (2000), the heated beam yields at the lower temperature of about 400°C and deflects gradually.

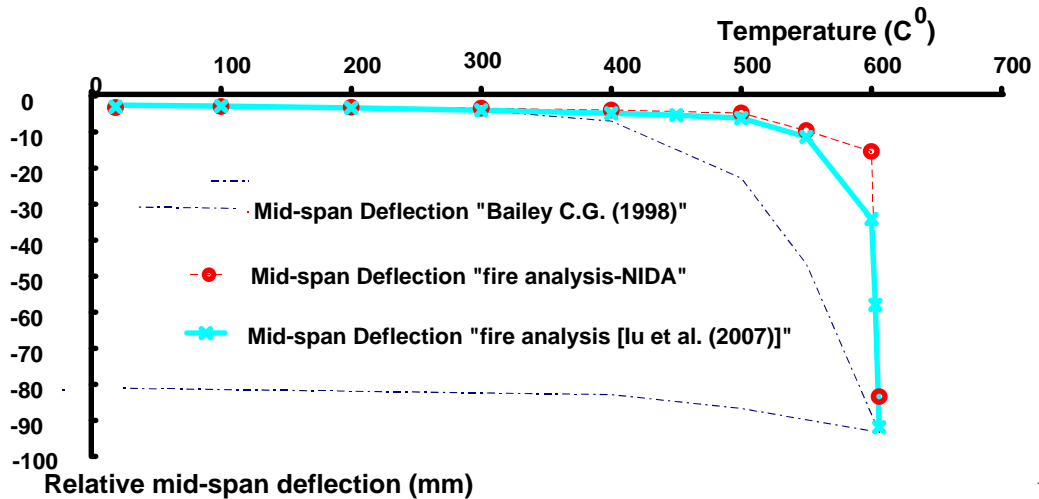


Figure 10: Mid-span deflection of beam at bay 5 subjected to compartment fire

5.3 Eight-storey plane frame at Cardington fire test

The eight-storey steel frame with concrete slab was constructed as a 3 bay deep and 5 bay wide structure at the Cardington fire test in UK where four fire tests were carried

out. The geometry and member section of the tested plane frame are also shown in Fig. 11. All the out-of-plane degrees of freedom of this frame are restrained and all connections are designed as rigid in the model for the present study. A distributed load of 5.48kN/m^2 was applied and fire was located at fourth floor. The maximum temperature is approximately 800°C . Although the temperature distribution across the beam at fourth floor level in the fire test is slightly non-uniform, the beam members are assumed to have been uniformly heated in the present computer modeling. The temperature distribution in the frame was described in details by many researchers, who include Plank et al. (1996), Kirby (1996,1997) and Sanad et al. (2000).

Kirby (1997) observed from the fire test that the inner primary beam deflected 293mm at 800°C . Also Iu et al. (2005) presented the numerical results of this eight-storey plane framed structure as a comparison in this fire study with a good correlation. The beam section in Iu et al. (2005) is based on transformed composite section, whereas the present fire study evaluates the deflection-temperature curve of unprotected steel frame as plotted in Fig. 11. Hence there is a discrepancy between the present analysis and the results by Iu et al. (2005) at normal and elevated temperature before the large deflection (see Fig. 12). According to the present numerical results, the heated beam begins to yield at 600°C gradually and behaves nonlinearly. The mid-span deflection of a beam of $610\times 229\times 101$ UB reaches 240mm ultimately, which is close to the results from the experimental results by Kirby (1997) and Sanad et al. (2000).

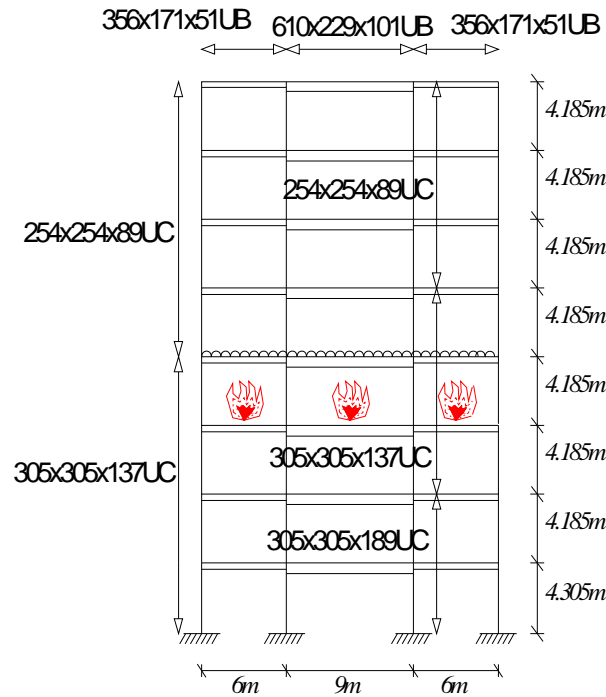


Figure 11: Geometry and member sections of eight-storey plane frame at Cardington fire test

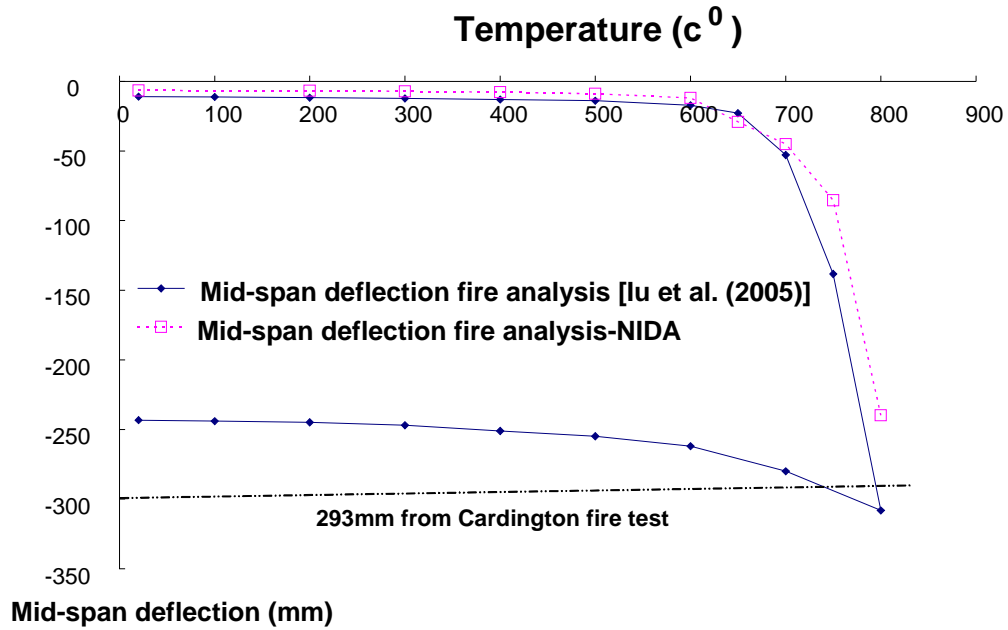


Figure 12: Mid-span deflection of 8-storey plane frame at Cardington fire test

6. CONCLUSIONS

The present structural fire study indicates that the nonlinear fire analysis by means of a second-order structural analysis software like NIDA can be efficiently carried out for the framed structures. In spite of its simplicity, the present fire study indicates that the structural behaviour of the framed structure under fire is generally in good agreement with different numerical and experimental results. Using the proposed method, the present simulation-based fire analysis can assist to understand the overall structural behaviour of complicated and large steel structures under fires. In addition, when the fire assessment of a framed structure is considered as a whole, fire protection of many elements in a steel building structure was found to be unnecessary in many cases because of the beneficial nonlinear effects such as centenary action in deformed beams. From the results reported here, the simulation-based fire analysis carries a high potential for effective and economical design.

An integrating fire system analysis and structural design can provide an improved understanding of the real behaviour of an entire structure attacked by fire. On the other hand, the prescriptive fire design codes, which only provide the design specifications for an isolated and individual member for its limit states behaviour, are approximate and over-conservative. From this observation, the present simulation-based fire approach is suitable to the performance-based fire safety design of an overall structural system. With the advance in computer speed, availability of personal computers and structural analysis software, there will be an increasing demand for simplified, robust and efficient nonlinear analysis methods for performance-based fire safety design of structures.

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